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July 1998

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Repair, Evaluation, Maintenance, and Rehabilitation Research Program

Accuracy of Estimating Compressive Strength of Deteriorated Concrete Seawall by Nondestructive Evaluation (NDE)

by A. Michel Alexander, WES

Prepared for Headquarters, U.S. Army Corps of Engineers

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	Problem Area		Problem Area
CS	Concrete and Steel Structures	EM	Electrical and Mechanical
GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

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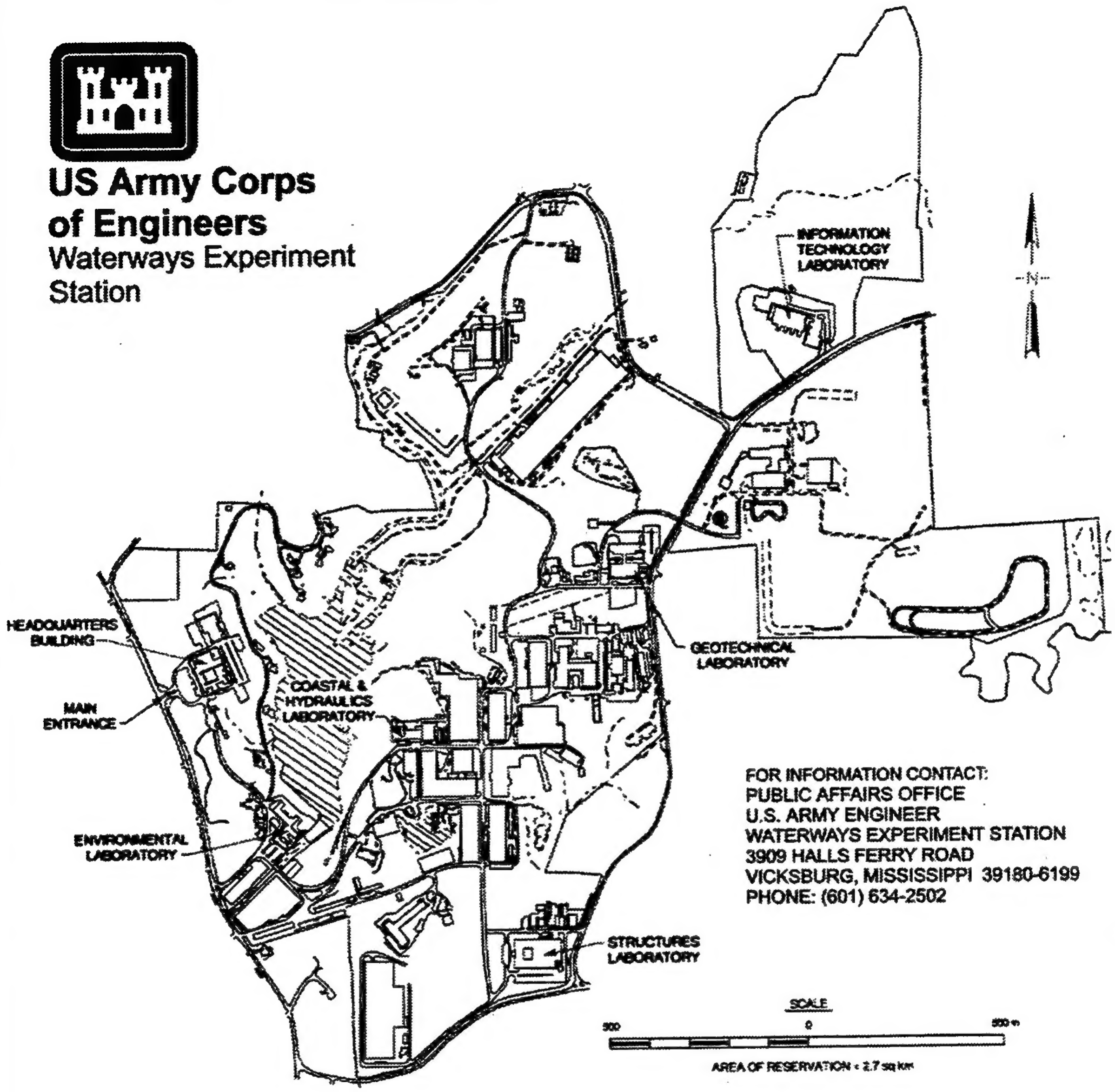

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Contents

Preface

The work described in this report was sponsored by Headquarters, U.S. Army Corps of Engineers (HQUSACE), as part of the Concrete and Steel Structures Problem Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program. The work was performed under REMR Work Unit 32638, "Nondestructive Evaluation Systems for Civil Works Structures," for which Mr. A. Michel Alexander, Structures Laboratory (SL), U.S. Army Engineer Waterways Experiment Station (WES), was Principal Investigator. Mr. M.K. Lee (CECW-EG) was the HQUSACE Technical Monitor.

Dr. Tony C. Liu (CERD-C) was the REMR Coordinator at the Directorate of Research and Development, HQUSACE. Mr. Harold C. Tohlen (CECW-O) and Dr. Liu served as the Overview Committee. Mr. William F. McCleese, SL, was the REMR Program Manager, and Mr. James E. McDonald, SL, was the Problem Area Leader.

The study was conducted under the general supervision of Dr. Bryant Mather, Director, SL, and Dr. Paul F. Mlakar, Chief, Concrete and Materials Division (CMD), SL.

The field work was performed in November 1990 on the seawall of Mandalay Bay Marina at Oxnard, CA. Mr. Jon Moore, Noble Consultants, Irving, CA, represented Oxnard. Oxnard furnished two men, a boat, and a truck and covered the expenses for drilling and shipping cores to WES. Messrs. Alexander and Willie E. McDonald, CMD, performed the field investigation.

Dr. Robert W. Whalin was Director of WES. COL Robin R. Cababa, EN, was Commander.

1 Introduction

Goals of Nondestructive Evaluation (NDE) Research

A primary concern for those organizations tasked with maintaining concrete structures is the assessment of such structures to ensure structural and operational safety. Destructive testing is not always a feasible alternative since it can be both time-consuming and expensive. When applicable, NDE can result in considerable savings. However, a standard evaluation method for estimating the compressive strength (CS) of concrete from NDE measurements does not currently exist in the United States, and new measurement standards are needed.

One of the goals of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program is to identify, develop, adapt, improve, and verify NDE technologies for accessing the condition of concrete structures in the field. The investigation described in this report was selected because it met the research goals of REMR Work Unit 32638, "Nondestructive Evaluation Systems for Civil Works Structures," and was of mutual interest to Oxnard, CA, and the U.S. Army Corps of Engineers. The work described herein was conducted by the Concrete and Materials Division, Structures Laboratory, U.S. Army Engineer Waterways Experiment Station (WES), with field support contributed by Oxnard.

Purpose of Investigation

This report describes an in situ NDE investigation of concrete pilaster support structures used to hold concrete panels in place for a marina seawall. The purpose of this study was to determine the accuracy of using NDE techniques to estimate the CS property of both good-quality and distressed concrete in the field. If the results showed a high correlation between CS and ultrasonic pulse velocity (UPV) or between CS and rebound hammer number (RN), then one of the two NDE (UPV or RN) methods could be used to

evaluate the complete seawall or other concrete structures and to estimate the CS in locations where the quality was unknown.

2 Background

History of NDE for In Situ Strength Measurements

The construction industry has not belonged to the *avant-garde* in the development of diagnostic techniques for evaluating the quality and condition of concrete structures. In fact, it lags far behind the medical, automotive, and aerospace industries in state-of-the-art NDE equipment and methods. However, in the last few years, various research organizations have promoted interest in the development of new and improved diagnostic equipment and techniques to aid in the repair and rehabilitation of the failing infrastructure of the Nation.

A time-honored method used by structural engineers to determine the integrity of a structure is to drill cores to test for CS. For obvious reasons (mostly related to cost), it is desirable to use NDE where possible to estimate CS. Both the UPV and RN methods are well-known for making comparative measurements over a structure to determine the location and extent of concrete having a variation in UPV or RN readings, or both. More often than not, the most important function of NDE has been to determine the uniformity of the condition or quality of the concrete in the structure. Although it is possible (a calibration must be performed) to estimate the CS from NDE measurements, this method has not been as common an undertaking in the United States as in Europe and Australia.

The use of NDE for estimating in situ CS has received much attention over the last few years in the United States. The American Concrete Institute (ACI) Committee 228 has produced "In-Place Methods to Estimate Concrete Strength" (ACI 228.1R-95). Malhotra and Carino (1991) recommend the UPV method for estimating in situ and precast concrete strength. Both RILEM (1972) and British Standards (1974) describe practices that use UPV measurements to estimate the in situ strength of a structure. Malhotra and Carino (1991) recommend combined-NDE techniques (such as UPV or RN with CS) as a viable alternative to destructive evaluation if a correlation exists; the key to this method lies in obtaining a calibration that establishes the correlation between CS and the corresponding NDE property of a few cores.

In practice, concrete quality is still commonly described in terms of a uniaxial CS measurement on a core or cylinder. For this reason, people still drill cores and measure CS rather than use UPV measurements to delineate regions that need maintenance and repair. (CS values are necessary when there is a need to determine the structural capacity of the structure.) In fact, the American Society for Testing and Materials (ASTM)[a] C 597 (ASTM 1991c) states that UPV measurements should not be used to estimate CS as the method exists now. However, this stipulation against using UPV measurements exclusively to estimate CS does not preclude its use in combination with coring to significantly improve estimations.

ASTM C 597 (ASTM 1991c) also advises against the exclusive use of UPV measurements to estimate the modulus of elasticity (although an analytical relationship between UPV and modulus of elasticity does exist), but it does state that corresponding pairs of UPV and modulus of elasticity values can be used to establish a UPV-modulus relationship.

Those responsible for maintaining the infrastructure of the nation may find it desirable to measure the CS of the concrete in their structure from time to time, whether to locate the presence and extent of deterioration or for other reasons. Recently, WES performed NDE for customers in Los Angeles, CA; Oxnard; and Whitney Point, NY. All three of these organizations elected to use UPV measurements in combination with coring rather than to core exclusively. Although most people will be fully satisfied with UPV measurements for the purpose of defining areas of acceptable and unacceptable concrete, this report contains information for those who, for whatever reason, intend to determine the CS of concrete in a structure. Even when the CS is not needed and a UPV survey is sufficient to locate the distressed regions, there are some who will have reservations about accepting UPV measurements alone as a viable method.

Oxnard Investigation

WES was contacted by a representative of Oxnard concerning visible signs of distress in the concrete pilaster support components of a seawall structure. The seawall is located in Channel Islands Harbor in the Mandalay Bay Marina, Oxnard. The problem area concerned a particular phase of the development of the seawall called the Boise-Cascade wall section. The other development phase, the Zurn wall, was not of immediate concern. A meeting was subsequently held with a city representative to discuss the problem in more detail. At that time, representatives from both WES and Oxnard agreed that they had a mutual interest in estimating the in situ concrete strength properties and, therefore, planned an NDE field investigation.

[a] All ASTM citations are given in the References at the end of the text.

The officials from Oxnard identified several pilasters that were experiencing severe cracking distress, and they decided to implement repair procedures for those pilasters. The first phase of the repair procedure involved: (a) deficient areas of concrete to allow observations of the interior deterioration, (b) determining the depth and degree of deterioration, and (c) attempting to identify the cause(s) of distress. (It was determined after the NDE investigation that the concrete was experiencing alkali-silica reaction.)

NDE measurements were performed prior to removing the deteriorated concrete in the pilasters. (A record of the mixture proportioning and materials was not available, and it was not possible to develop the correlation curves from UPV measurements and CS tests on cylinders made from that mixture). The effectiveness of the NDE methods to determine the condition of the concrete was then determined by evaluating the CS of drilled concrete cores, plotting correlation curves, and calculating correlation coefficients. The degree of correlation obviously depended on getting the proper number, size, and quality of cores at a given location to properly represent the average condition of the evaluated area. The pilasters targeted for NDE included two located near Victoria Street, two near Monaco Street, and one near Napoli Street. An additional pilaster near Napoli Street was later added to the study. The names given for the measurement sites represent the designated street names in the Mandalay Bay Marina area.

3 Field Work on Concrete Pilasters

Measurement of Field UPV

Method of evaluation - ASTM C 597

UPV evaluations were conducted as established by ASTM C 597 (ASTM 1991c). The measurement apparatus included a V-meter and the accompanying 54-kHz transducers for signal transmission and reception for measuring the through-transmission time in microseconds for a pulse to travel in the concrete. It is well-known in the NDE community that the quality of concrete is related to the UPV. In fact, there is an analytical expression that relates the Young's modulus to the square of the UPV. On the contrary, an analytical expression does not exist that relates the CS to the UPV. Although a definite relationship exists between the latter quantities, it must be determined empirically on a given concrete mixture by calibration.

Type of structure and setting

The pilasters (and panels), comprising the seawall, were of precast concrete construction. A typical pilaster is shown in Figure 1. The ends of the panels of the seawall were butted against the pilasters at the time of construction (circa 1968, approximately). Both components were separated by some type of foam material provided at each of the panel/pilaster interfaces to provide joint separation for the purpose of thermal expansion. The major part of the foam material had deteriorated since the time of construction. The pilasters that contained a 203- by 203- by 25.4-mm (8- by 8- by 1-in.) embedded steel plate were anchored to a 38.1-mm (1.5-in.)-diam steel rod attached to the "deadman" weight. This weight was located 6.6 m (21.5 ft) from the wall within the backfill material (soil).

The panels were constrained on the soil side of the marina by the backfill pressures. WES conducted UPV evaluations on six target pilasters. Five of

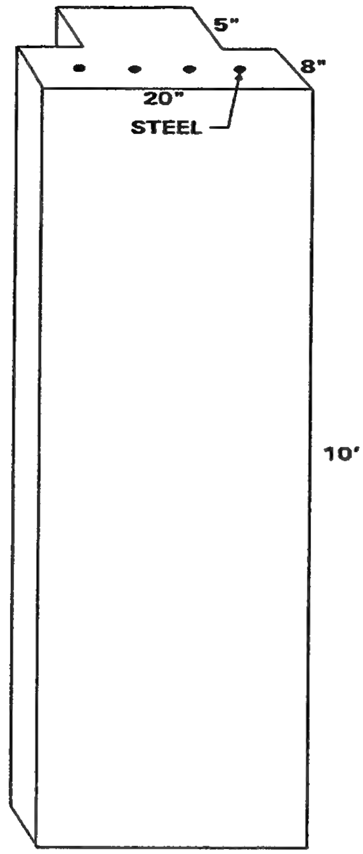

Figure 1. Pilaster drawing. (To convert inches to millimetres, multiply by 25.4; to convert feet to metres, multiply by 0.3048)

these pilasters showed evidence of varying levels of cracking and deterioration. Generally, there were wide discrete cracks spaced a fraction of an inch (millimetre) to a few inches (millimetres) apart rather than small uniformly continuous microcracks distributed across the surface. The sixth pilaster evaluated was chosen for a control, because it showed no visual evidence of cracking or other signs of deterioration.

Measurement scheme

The UPV survey was performed in two stages: first, a broad evaluation of the total pilaster was performed using a large vertical separation between measurement points to find the general location of the best and worst quality

concrete, and second, a more narrow survey of the two selected parts of the pilaster was made using a smaller vertical separation distance between measurement points to determine more accurately the UPV of the concrete in both locations. The best concrete has the highest UPV, and the worst concrete has the lowest UPV for a given concrete mixture. The initial UPV measurements were made at 0.3048-m (1-ft)-vertical intervals along the length of the pilasters with the first measurement taken 0.15 m (0.5 ft) from the top. The UPV measurements were taken horizontally across the width of the pilasters, a distance of approximately 508 mm (20 in.). As shown by Figure 1, the front surface of the pilasters stands out from the front surface of the concrete panels by 203 mm (8 in.), permitting space for locating the two 50.8-mm (2-in.)-diam UPV transducers on both sides of the pilaster. As a result, the UPV values obtained should, in theory, indicate the average condition of the concrete through the 508-mm (20-in.) width. Later, after finding the approximate location of the best and worst concrete in the pilaster, the measurement separation distance was shortened to 76 mm (3 in.) for a more discriminating survey.

Field conditions and UPV measurements

As expected, in the areas where the pilasters were cracking and the deterioration was visibly evident, lower UPV values were obtained. All of the pilasters evaluated showed higher UPV values at bottom locations (equal to or greater than 1.5 m (5 ft) below the top of the pilaster).

During the UPV measurements, the transducers were normally positioned near the center of the 203-mm (8-in.)-wide vertical surface on either side of the pilasters. However, in a couple of cases, there were cracks near the center of the side faces running parallel with the front vertical edges of the pilaster. In these cases, the normal positioning of the transducers near the centers of the sides did not properly reflect the average condition of the pilaster. Therefore, for such situations, it was decided to perform UPV measurements both in front of and behind the crack for a better average representation. The results of the UPV measurements for the broad survey are given in Tables 1-6 for each of the pilasters. The same data are plotted in bar graph form in Figures 2-7.

Except for a couple of cases, the pilasters had deterioration, predominantly, as evidenced by the visible cracks, in the vicinity of the 203- by 203- by 25.4-mm (8- by 8- by 1-in.) embedded-steel plates. Each pilaster has a plate located at a vertical position of 1.2 m (4 ft) from the top. Several pilasters showed cracks near the top. Generally, the cracks were a few feet in length, spaced a few inches (millimetres) apart, and ran in a vertical direction on the front face of the pilaster. Initially, it appeared that the crack might be caused by the expansion of corroded reinforcement bars that traversed the 3.0-m (10-ft) length of the pilasters, since the steel and surface cracks were parallel in direction. However, the reinforcement bars were in a noncorroded condition when the repair crew removed the deteriorated concrete.

Table 1
Field Ultrasonic Pulse Velocities - Victoria, North

Position from Top of Pilaster, ft	Pulse Velocity, ft/sec
0.5	11,922
1.5	12,350
2.5	10,079
3.5	11,254
4.5	11,768
5.5	14,318
6.5	14,668
7.5	14,148
8.5	15,022

Note: To convert feet to metres, multiply by 0.3048.

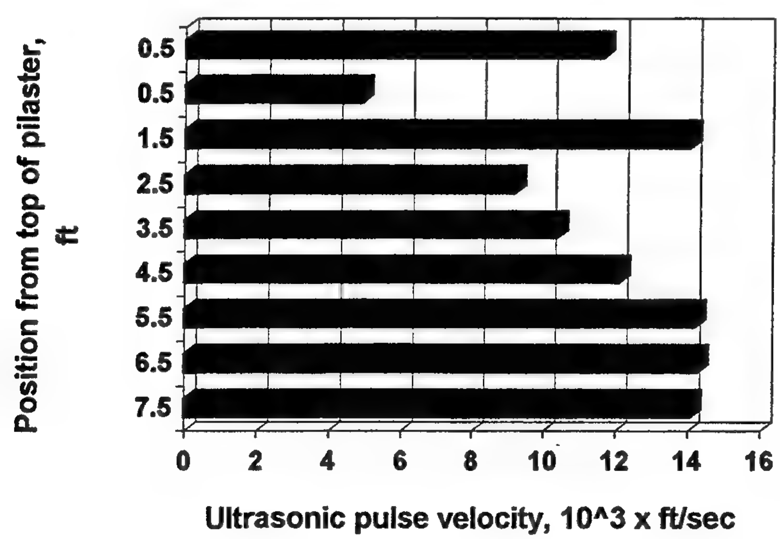

Figure 2. Victoria, north, pilaster UPV. (To convert feet to metres, multiply by 0.3048)

Table 2
Field Ultrasonic Pulse Velocities - Victoria, South

Position from Top of Pilaster, ft	Pulse Velocity, ft/sec
0.5	11,590 (front)
0.5	4,839 (back)
1.5	14,028
2.5	9,151
3.5	10,327
4.5	12,065
5.5	14,169
6.5	14,241
7.5	14,039

Note: To convert feet to metres, multiply by 0.3048.

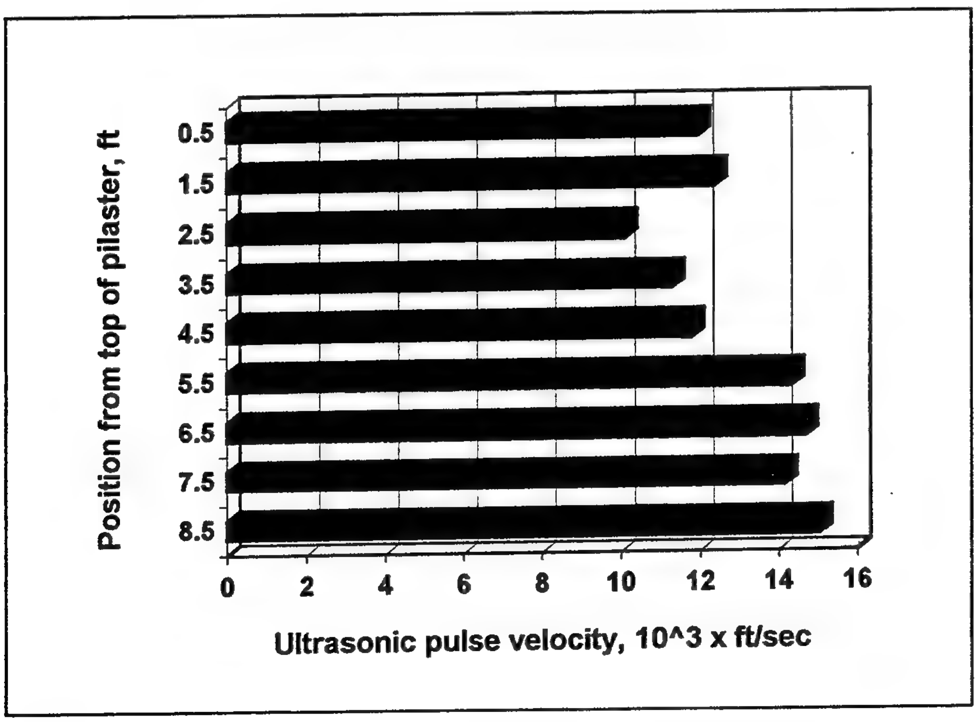

Figure 3. Victoria, south, pilaster UPV. (To convert feet to metres, multiply by 0.3048)

Table 3
Field Ultrasonic Pulse Velocities - Monaco, North

Position from Top of Pilaster, ft	Pulse Velocity, ft/sec
0.5	13,123
1.5	13,764
2.5	9,096
3.5	7,512
4.5	8,195
5.5	13,425
6.5	14,636
7.5	14,623

Note: To convert feet to metres, multiply by 0.3048.

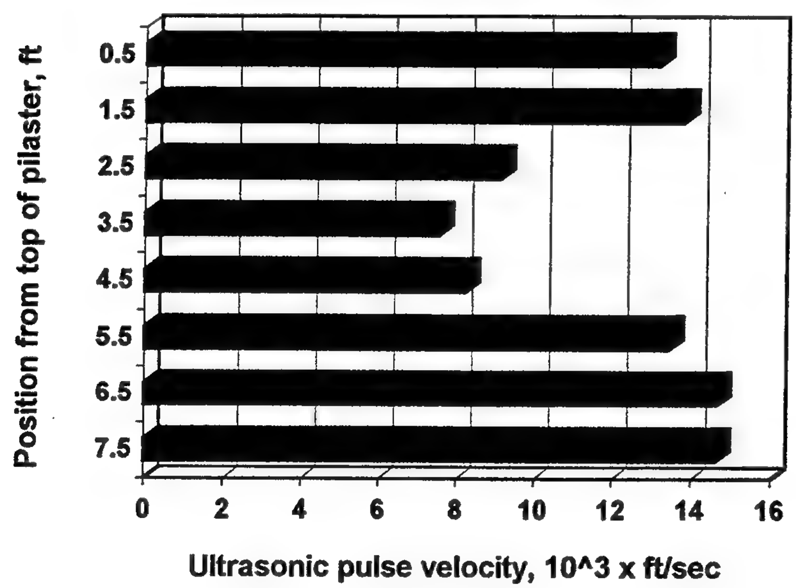

Figure 4. Monaco, north, pilaster UPV. (To convert feet to metres, multiply by 0.3048)

Table 4
Field Ultrasonic Pulse Velocities - Monaco, South

Position from Top of Pilaster, ft	Pulse Velocity, ft/sec
0.5	13,895
1.5	14,081
2.5	9,067
3.5	7,618
4.5	4,107
5.5	10,875
6.5	14,613
7.5	14,564

Note: To convert feet to metres, multiply by 0.3048.

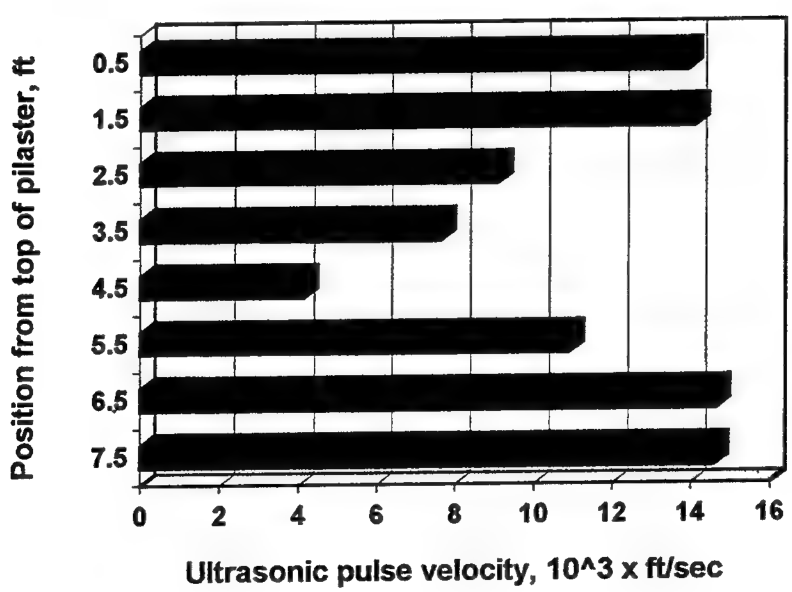

Figure 5. Monaco, south, pilaster UPV. (To convert feet to metres, multiply by 0.3048)

Table 5
Field Ultrasonic Pulse Velocities - Napoli, North[a]

Position from Top of Pilaster, ft	Pulse Velocity, ft/sec
0.5	13,021
1.5	13,855
2.5	14,004
3.5	14,460
4.5	13,478
5.5	13,969
6.5	13,697
7.5	14,016
8.5	13,720

Note: To convert feet to metres, multiply by 0.3048.
[a] Control pilaster, no evident deficiencies.

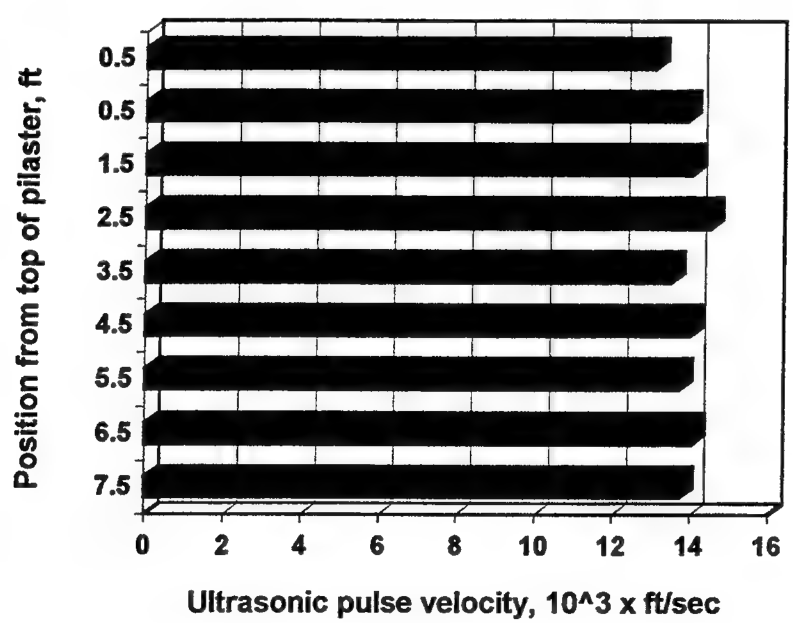

Figure 6. Napoli, north, pilaster UPV. (To convert feet to metres, multiply by 0.3048)

Table 6
Field Ultrasonic Pulse Velocities - Napoli, South

Position from Top of Pilaster, ft	Pulse Velocity, ft/sec
0.5	13,226
1.5	13,759
2.5	13,090
3.5	13,968 (front)
3.5	10,724 (back)
4.5	13,710
5.5	13,653
6.5	13,642
7.5	14,121

Note: To convert feet to metres, multiply by 0.3048.

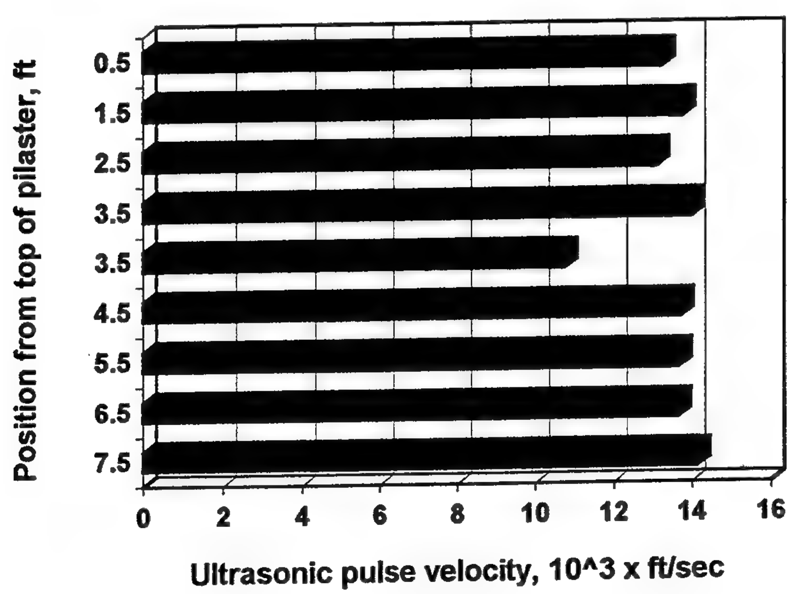

Figure 7. Napoli, south, pilaster UPV. (To convert feet to metres, multiply by 0.3048)

WES designated two locations on each of the pilasters for core extractions. Three cores were drilled in a horizontal line at each of the two locations rather than one core to make a better average determination of the the concrete CS across the 508-mm (20-in.) distance. As previously mentioned, in these two locations on each pilaster, additional UPV measurements were taken at 76-mm (3-in.) vertical intervals along the height of the pilasters to better determine the average UPV of the concrete in that area and define the condition profile. The UPV values are not shown. The extraction plan was to remove samples at only the two locations where the concrete had the highest and lowest UPV and hence obtain the highest and lowest CS values for the cores. The idea was to derive an improved correlation curve, because it would span the total range from the lowest UPV and its corresponding CS to the highest UPV and its corresponding CS rather than merely covering a short segment of the total curve.

Measurements of Rebound Number (RN)

Method of evaluation - ASTM C 805

RNs were measured in accordance with ASTM C 805 (ASTM 1991e) to ascertain indications of the surface hardness. The RN evaluation method requires that 10 readings be taken within an area of a 152-mm (6-in.)-diam circle and that no two measurements be taken closer than 25.4-mm (1-in.) apart. Each of the 10 impact numbers are required to be within 7 units of the overall average. Otherwise, a number outside this value must be discarded and the average computed using the nine remaining measurements. In the event two or more evaluation numbers are outside this value, the entire set of readings should then be discarded. The RN evaluations were performed using the standard Schmidt-hammer apparatus.

Measurement scheme and setting

The RN measurements were taken at 0.6-m (2-ft)-vertical intervals along the height on the front faces of the target pilasters with the first evaluation position being 0.46m (1.5 ft) from the top. Since the RN evaluations are dependent on the surface condition of the concrete, the establishment of proper comparative relationships between the readings must be considered with respect to a fixed height on the pilasters. The reason for this surface difference is due to the varying exposure to water. At various heights, the pilaster surfaces are exposed to underwater conditions for extended periods of time depending on tide fluctuations. Thus, comparisons of the surfaces relative to each pilaster should be based upon the results taken at a fixed height.

During measurements for RN, surfaces showing visible signs of deterioration were indicated by muffled impact sounds and lower RN. In fact, in some of the areas, the impact of the Schmidt hammer actually crushed the concrete

surface. The results of the average RN can be seen in Table 7 for each of the pilasters.

Table 7
Rebound Number, Average Readings

Location from Top	1.5 ft	3.5 ft	5.5 ft	7 ft
Victoria, north	40.8	31.0	32.8[a]	36.8
Victoria, south	39.1	33.0	28.6	36.2
Monaco, north	36.6	27.4	27.8	38.9
Monaco, south	38.6[a]	26.2	20.5[b]	31.3[a]
Napoli, north	46.1	43.4	42.2	38.3
Napoli, south	44.0	37.0	39.8[b]	37.0

Note: To convert feet to metres, multiply by 0.3048.
[a] One measurement was seven units outside the average.
[b] Two measurements were seven units outside the average.

4 Core Extractions

Coring Scheme

Oxnard agreed to handle the drilling and shipping of cores to WES. WES marked the location for the crew to drill 36 cores from the six pilasters. Six cores were taken from each pilaster. Three of the cores were in a zone considered to be the highest quality concrete as determined by the highest UPV, and the other three were in a zone considered to be the lowest quality concrete for that pilaster as determined by the lowest UPV. Since the field UPV measurements represented an average condition over a 508-mm (20-in.) path length, it was desirable to obtain more than one core to get a better average of the CS of the concrete at that location. The cores were drilled horizontally with a diamond bit as required by ASTM C 42 (ASTM 1991b).

Method of Evaluation - ASTM C 42

After the cores were received at WES, they were screened and prepared for evaluation according to ASTM C 42. Of the 36 cores received, only 22 were accepted for laboratory evaluation. Most of the cores at the Victoria site were lost for two reasons: first, the drill bit was too small in diameter to produce acceptable cores, and second, the concrete was too deteriorated to permit the recovery of sound cores. Once researchers at WES realized that the original cores were being drilled with a hand-held drill with a bit having too small a diameter (43.2 mm (1.7 in.)), they recommended that the contractor obtain another drill rig with a larger-diameter (57 mm (2.25 in.)) bit that could be firmly attached to the structure. Consequently, the contractor was able to recover three adequate cores from the bottom location on the Victoria north pilaster by using the larger diameter bit. Also, the larger diameter bit was used at the Monaco and Napoli sites.

The location of the accepted and rejected cores are shown in Figures 8, 9, and 10. Some of the remaining cores were of generally poor condition or of unacceptable dimensions for performing laboratory evaluation. All of the cores accepted met the "shall be" criteria for the length-to-diameter (l/d) ratio,

Victoria North

TL	TC	TR
*	*	*
BL	**BC**	**BR**
-	-	-

Victoria South

TL	TC	TR
*	*	*
BL	**BC**	**BR**
*	*	*

- Acceptable core
* Unacceptable core

TL - Top Left
TC - Top Center
TR - Top Right

BL - Bottom Left
BC - Bottom Center
BR - Bottom Right

Figure 8. Location of acceptable cores, Victoria, north and south

Monaco North

TL	TC	TR
-	*	-
BL	**BC**	**BR**
*	*	*

Monaco South

TL	TC	TR
-	-	-
BL	**BC**	**BR**
-	?	*

- Acceptable core
* Unacceptable core
? Location in question

TL - Top Left
TC - Top Center
TR - Top Right

BL - Bottom Left
BC - Bottom Center
BR - Bottom Right

Figure 9. Location of acceptable cores, Monaco, north and south

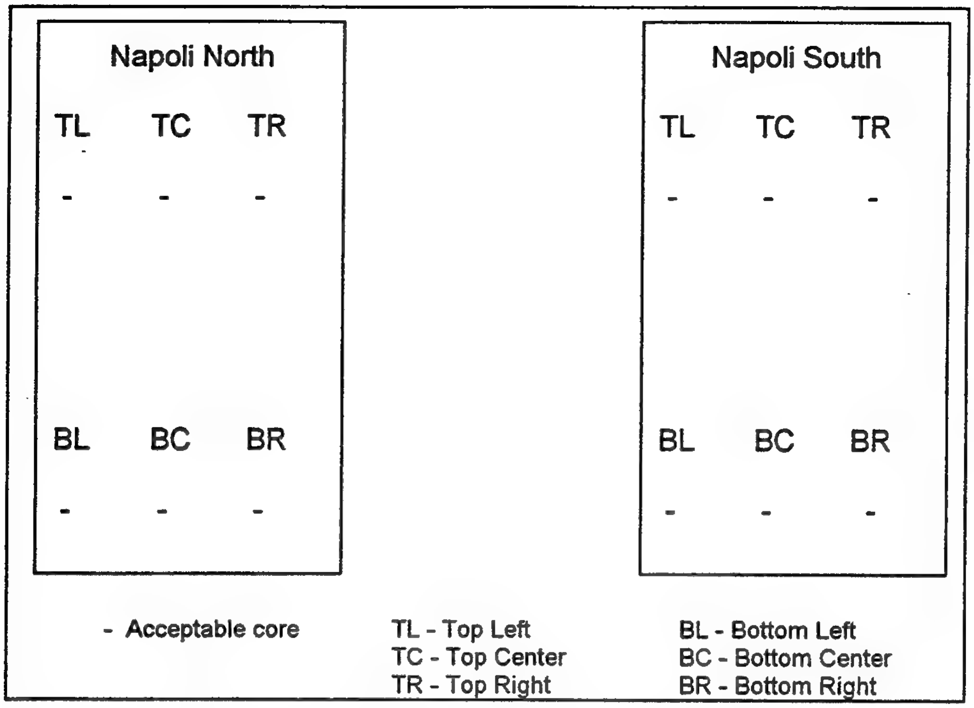

Figure 10. Location of acceptable cores, Napoli, north and south

but some did not meet the "preferable" criteria laid down in the ASTM C 42 evaluation method. The evaluation method recommends that the diameter of the core be at least twice the nominal maximum dimension of the coarse aggregate in the concrete and preferably three times the nominal maximum dimension. For 19-mm (3/4-in.) aggregate, this would be 63.5 mm (2.5 in.). Also, when UPV measurements are made, 50.8 mm (2 in.) is a minimum diameter for obtaining proper results. All of the 43-mm (1.7-in.)-diam specimens were rejected for CS evaluations. The l/d ratio of some of the 57-mm (2.25-in.)-diam cores evaluated were less than 1.94, which in accordance with ASTM C 39 (ASTM 1991a) requires a correction factor for calculating the CS. However, as mentioned earlier, ASTM C 42 (ASTM 1991b) suggests that all cores should have an l/d ratio of 2 or greater for evaluating CS. The CS given in Table 8 reflect such corrections where necessary.

Preparation of Cores

The cores as drilled are shown in Figures 11 and 12. Most of the cores from the Victoria site failed to meet ASTM C 39 criteria and were not analyzed. In accordance with ASTM C 42, the ends of all cores were made flat by sawing the end surfaces perpendicular to the longitudinal axis. The diameter of a number of cores did not meet the requirement that the end measurements not depart more than 2.5 mm (0.1 in.) from the mean diameter

Table 8
Laboratory UPV and CS for Individual Cores

Pilaster Site	Core Site	Core Location ft	Pulse Velocity ft/sec	Compressive Strength, psi
Victoria, north	Top right	3.50	–	–
Victoria, north	Top center		–	–
Victoria, north	Top left		–	–
Victoria, north	Bottom right	6.00	14,352	5,917
Victoria, north	Bottom center		14,286	5,779
Victoria, north	Bottom left		13,726	5,071
Victoria, south	Top right	0.50	–	–
Victoria, south	Top center		–	–
Victoria, south	Top left		14,323 [a]	–
Victoria, south	Bottom right	6.00	14,865 [a]	–
Victoria, south	Bottom center		15,217 [a]	–
Victoria, south	Bottom left		13,554 [a]	–
Monaco, north	Top right	0.50	14,471	4,321
Monaco, north	Top center		–	–
Monaco, north	Top left		14,045	3,213
Monaco, north	Bottom right	3.00	10,325 [b]	–
Monaco, north	Bottom center		15,350 [b]	–
Monaco, north	Bottom left		9,259 [b]	–
Monaco, south	Top right	0.50	14,181	4,672
Monaco, south	Top center		14,228	4,854
Monaco, south	Top left		14,530	4,426
Monaco, south	Bottom right	5.00	9,033	1,534
Monaco, south	Bottom center [c]		16,089	5,063
Monaco, south	Bottom left		–	–
Napoli, north	Top right	1.00	13,189	5,436
Napoli, north	Top center		13,441	4,368
Napoli, north	Top left		13,109	4,433

(Continued)

Note: To convert feet to metres, multiply by 0.3048.
To convert pounds (force) per square inch to megapascals, multiply by 0.006894757.
[a] Diameter too small to be acceptable.
[b] Too short in length.
[c] Doubt about correct location.

Table 8 (Concluded)				
Pilaster Site	**Core Site**	**Core Location ft**	**Pulse Velocity ft/sec**	**Compressive Strength, psi**
Napoli, north	Top left	1.28	13,333	2,718
Napoli, north	Bottom right	3.25	12,784	3,094
Napoli, north	Bottom center		13,837	3,068
Napoli, north	Bottom left		12,861	4,260
Napoli, south	Top right	1.00	12,440	5,286
Napoli, south	Top center		13,122	2,764
Napoli, south	Top left		12,927	4,607
Napoli, south	Bottom right	3.25	14,315	4,501
Napoli, south	Bottom center		14,340	4,272
Napoli, south	Bottom left		13,359	4,653

of the specimen. Also, another requirement is that the length of the core be at least twice the diameter of the core, and many cores failed to meet that criterion. The ends of the specimens were capped according to evaluation method requirements in ASTM C 617 (ASTM 1991d). The analyses performed in the laboratory included CS and UPV evaluations.

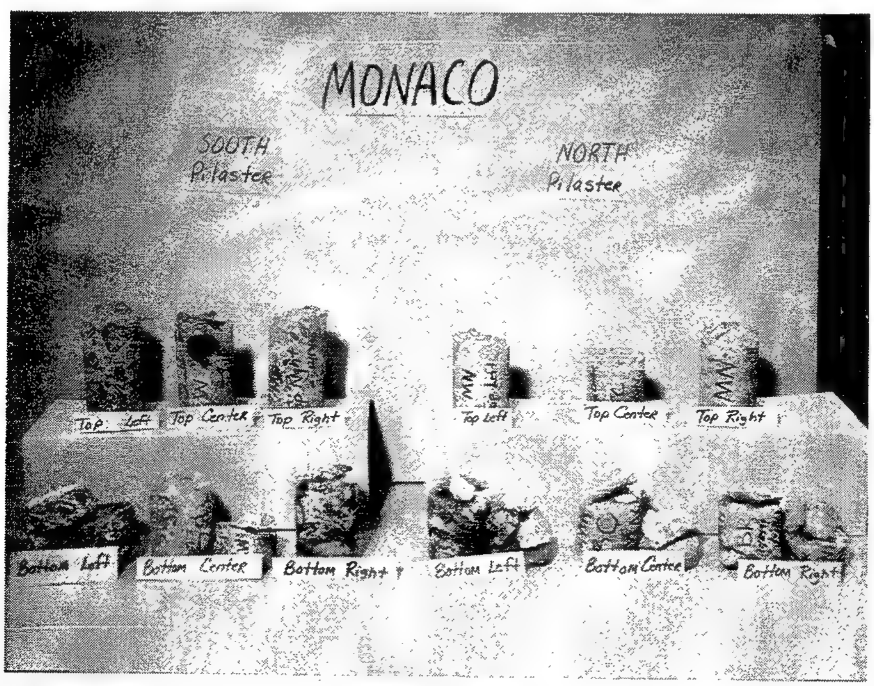

Figure 11. Drilled cores from Monaco site

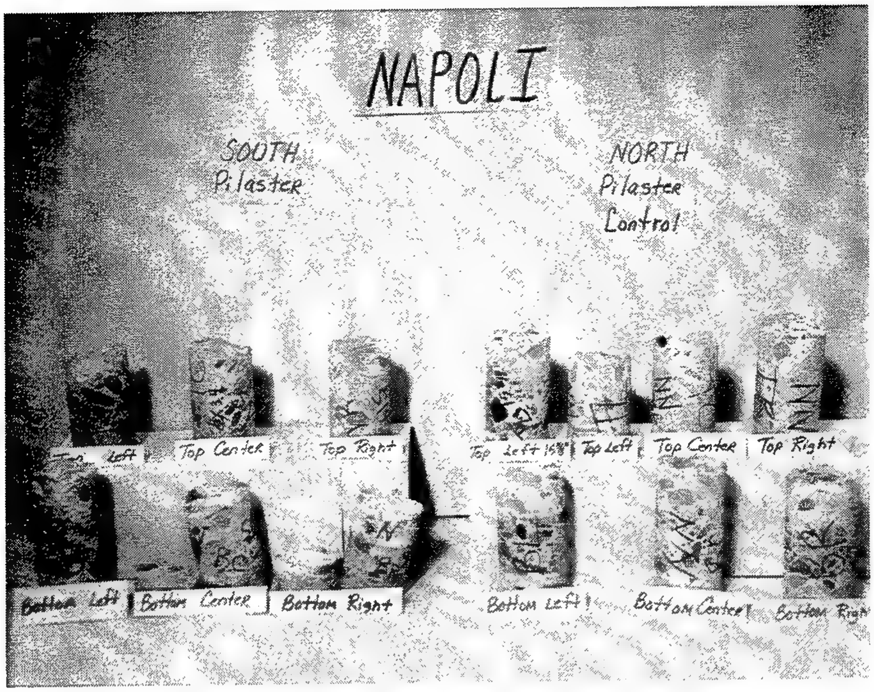

Figure 12. Drilled cores from Napoli site

5 Laboratory Evaluations

Compressive Strength (CS) Evaluations

WES evaluated the cores for CS in accordance with ASTM C 39 (ASTM 1991a). The results of the CS evaluations showed values ranging from 10.6 to 40.8 MPa (1,534 to 5,917 psi). UPV and CS laboratory evaluation results for the larger diameter cores (≈57 mm (2.25 in.) are given in Table 8.

UPV evaluations

The UPV evaluations were conducted in accordance with ASTM C 597 (ASTM 1991c) for the cores of acceptable condition and dimensions. The range of UPV readings for these cores was from 2,753 to 4,904 m/sec (9,033 to 16,089 ft/sec). Although the validity of UPV measurements for assessing the in situ condition (not CS) of concrete has been confirmed and documented by others, there are many variables relating to the composition and condition of the concrete that must be taken into account to properly interpret UPV results. However, it has been generalized that for normal mass concrete (approximately 150 pcf [a]), the condition of the concrete can be rated as shown in Table 9 (Leslie and Cheesman 1949).

Even though laboratory studies have shown a good correlation between the CS and UPV, these correlations were performed on cylinders and not on cores. One would expect more scatter in the correlation curve from data on drilled cores than from data on laboratory-cast cylinders. Cylinders have a shape that approximates a perfect circle, whereas drilled cores are generally not shaped that precisely. Also, cores undergo stresses from the drilling operation that may affect the mechanical properties of the concrete.

With 19-mm (3/4-in.) aggregate, a specimen should be at least 50.8 mm (2 in.) in diameter or more to get a proper average of its heterogeneous condition. Ideally, it should have an l/d ratio of two, but a 1- to 1-ratio is

[a] To convert pounds (mass) per cubic foot to kilograms per cubic metre, multiply by 16.04.

permitted. The average values of CS of the three cores per location are shown in Table 10.

Table 9
Suggested Pulse Velocity Ratings for Concrete[a]

Ultrasonic Pulse Velocity, ft/sec	Quality of Concrete
>15,000	Excellent
12,000 - 15,000	Good
10,000 - 12,000	Questionable
7,000 - 10,000	Poor
<7,000	Very Poor

Note: To convert ft/sec into m/sec, multiply by 0.3048.
[a] After Leslie and Cheesman (1949).

Table 10
UPV and CS Correlation Data

Pilaster Site	Location from Top of Pilaster, ft	Field UPV from narrow survey, ft/sec	Average Length of Cores, in.	Average CS, psi	Average Laboratory UPV, ft/sec
Victoria north, top (VNT)	3.5	9,320	<1.0	–[a]	–[a]
Victoria north, bottom (VNB)	6.0	14,495	3.2	5,590 (3)[c]	14,120 (4)
Victoria south, top (VST)	0.5	4,840	1.2	–[b]	14,325 (1)
Victoria south, bottom (VSB)	6.0	14,200	2.8	–[b]	14,545 (3)
Monaco north, top (MNT)	0.5	13,125	2.7	3,765 (2)	14,030 (3)
Monaco north, bottom (MNB)	3.0	8,570	1.7	–[a]	9,260 (1)
Monaco south, top (MST)	0.5	13,805	3.4	4,650 (3)	14,315 (3)
Monaco south, bottom (MSB)	5.0	9,770	2.2	1,535 (1)	9,035 (1)
Napoli north, top (NNT)	1.0	13,020	3.6	4,240 (4)	13,270 (4)
Napoli north, bottom (NNB)	3.3	14,460	4.1	3,474 (3)	13,160 (3)
Napoli south, top (NST)	1.0	13,225	3.3	4,220 (3)	12,830 (3)
Napoli south, bottom (NSB)	3.3	13,205	3.2	4,475 (3)	14,005 (3)

Note: To convert feet to metres, multiply by 0.3048.
To convert inches to millimetres, multiply by 25.4.
To convert pounds (force) per square inch to megapascals, multiply by 0.006894757.
[a] Length too short.
[b] Diameter too small.
[c] The number of cores averaged are given in parentheses.

6 Analysis of Data

RN Measurements

The correlation of the average CS with the average RN at a given height on a pilaster is shown in Figure 13. The correlation coefficient was only 0.61. See Table 7 for the RN values and Table 10 for the CS values. Likely, the wide variability in the thickness of the marine deposits on the concrete surface, due to tide fluctuations, may well have prevented getting a proper correlation. On the lower part of the pilasters where the tide remained for a longer time, the deposits were thicker. Therefore, the rebound hammer could yield a false indication of the structural integrity at the lower elevations. According to the evaluation method, the surface should be ground to a depth of 6.4 mm (0.25 in.) below the surface for proper measurement conditions. To keep from marring the appearance of the structure, the WES team ground the marine deposits only to a depth flush with the surface of the concrete. This was done with a manual grinding stone. A seawall is probably one of the few types of structures where an attempt should not be made to relate the internal condition of the concrete to the surface condition because of marine deposits.

Correlation of Field and Laboratory Data

The correlation curve in Figure 14 shows that the average length of the three cores for a given height location on the pilasters that can be extracted from the structure is related to the field UPV of the concrete. The correlation coefficient is 0.86. The data are shown in Table 10. Poorer quality concrete is less able to resist the forces of drilling, and the core tends to break off before a full length can be obtained. Obviously, discontinuities that cut through the axis of the core either fully or partially prevent the removal of the full-length core. This reinforces the theory that one should not expect good core recovery in locations of extreme damage or deterioration.

The correlation curve given in Figure 15, relating the average laboratory UPV of the three cores per location to the field UPV, shows that a minimum of three cores is sufficient to yield a satisfactory correlation between the average

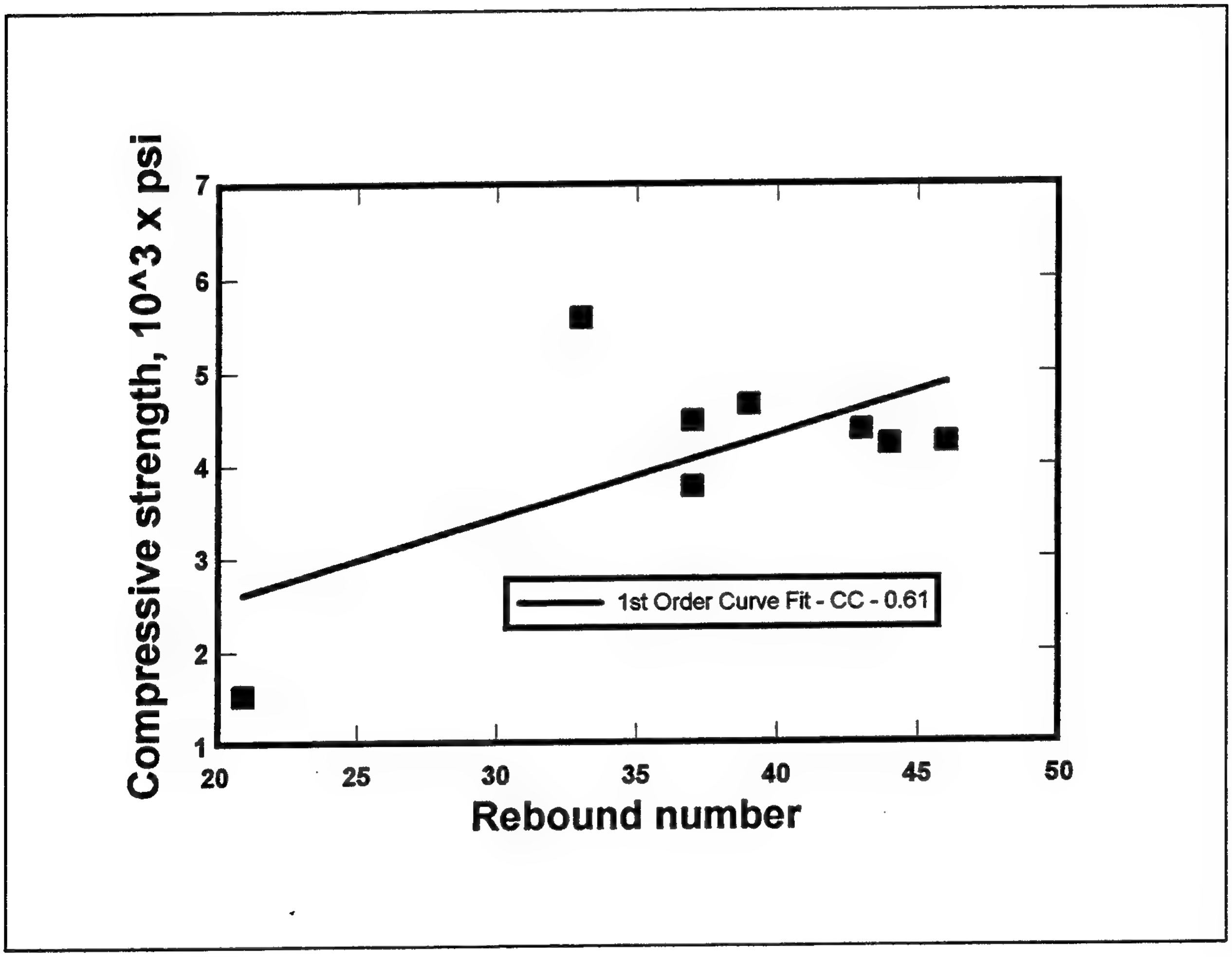

Figure 13. Correlation of average CS and average RN. (To convert pounds per square inch to megapascals, multiply by 0.006894757)

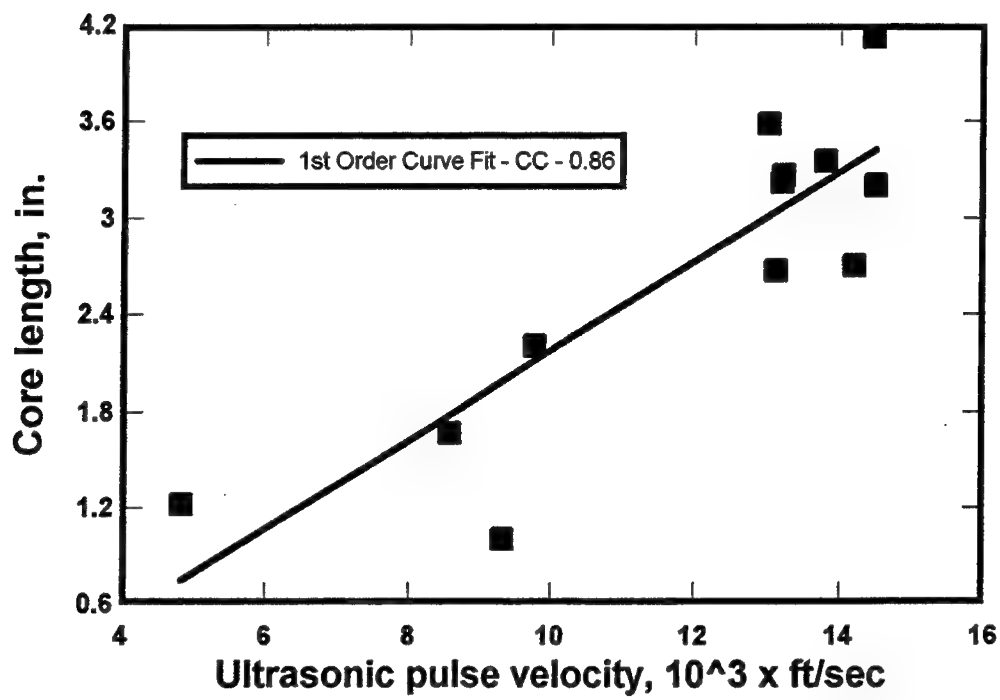

Figure 14. Correlation of core lengths averaged and field UPV. (To convert inches to millimetres, multiply by 25.4; to convert feet to metres, multiply by 0.3048)

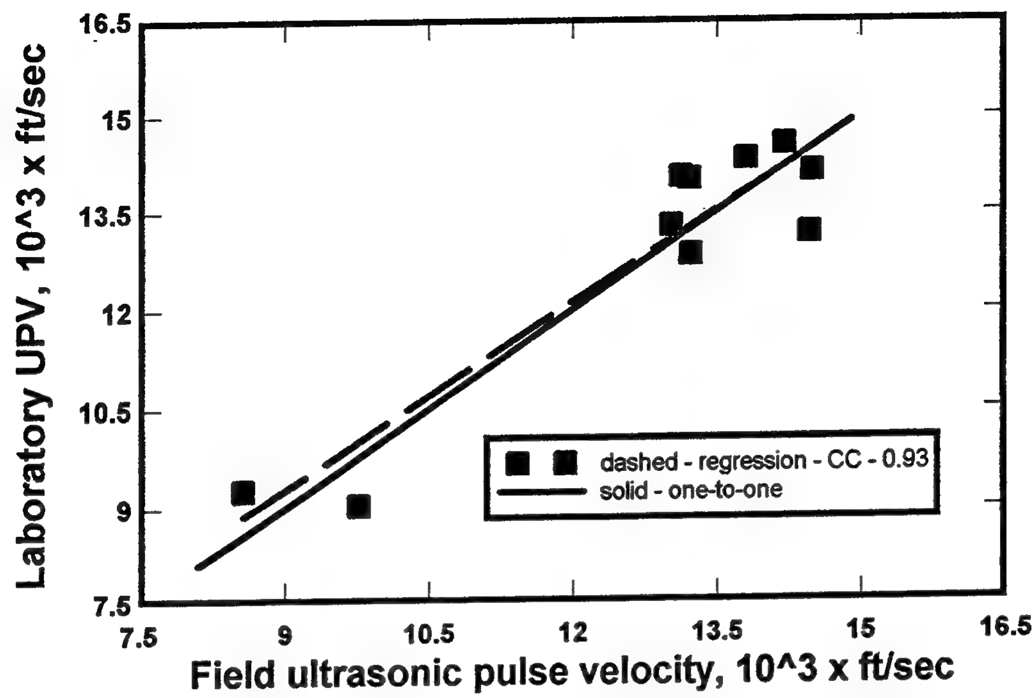

Figure 15. Correlation of average laboratory UPV and field UPV. (To convert feet to metres, multiply by in 0.3048)

laboratory UPV of the cores and the field UPV. The correlation coefficient is 0.93. The data can be seen in Table 10. The dashed line represents a linear regression of the data using Mendel's method, and the solid line represents equal laboratory and field UPV. One or two cores would not have been sufficient to obtain a correlation between the laboratory UPV and the field UPV because of the wide variation in the condition of adjacent cores at a given location. Also, the correlation shows that UPV anisotrophy is not a significant factor since the field and laboratory measurements, which were taken orthogonal to each other, correlate well.

The correlation in Figure 16 shows the relationship between the CS of each individual core and the laboratory UPV on that core. The data values were taken from Table 8. The correlation coefficient is only 0.55. This indicates that the UPV of any one core is certainly not very representative of the CS of that core. A larger-diameter core might have improved the correlation coefficient, but the close spacing of the reinforcement steel discouraged the use of larger drill bits.

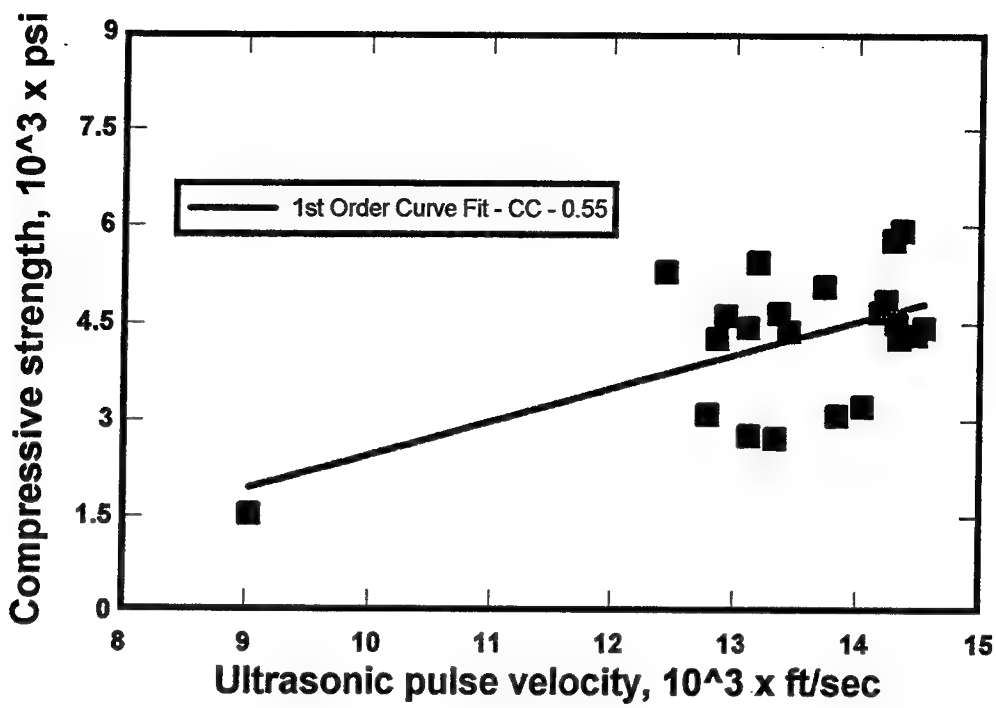

Figure 16. Correlation of individual core CS and laboratory UPV. (To convert pounds per square inch to megapascals, multiply by 0.006894757)

The curve shown in Figure 17 is the correlation of the average CS and the average laboratory UPV of the three cores per location. The correlation coefficient is 0.89 and much improved over Figure 16. The data values were taken from Table 10. This sets an upper limit on the accuracy of correlation expected of CS with the field UPV since the laboratory UPV measurements would likely be more accurate than the field measurements.

The correlation shown in Figure 18 shows the relationship between the average CS of three cores to the corresponding field UPV at the same locations. This correlation also indicates that more than one core taken at a location is important in improving the accuracy of estimation. The correlation coefficient is 0.84, a significant improvement over the unaveraged CS given in Figure 16. The CS of the cores can vary considerably over the short horizontal distance of a few inches between core locations. These results indicate that the UPV technique can determine a better average condition of the concrete at a location than the UPV or CS information from one core. Limited sampling by coring may create a poor correlation with UPV.

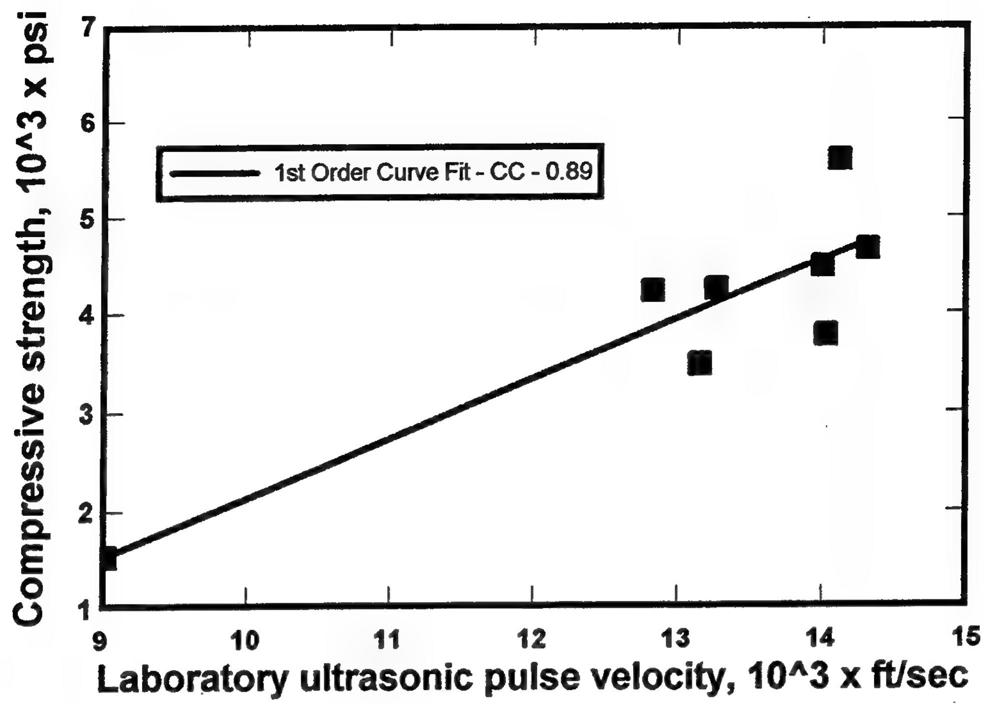

Figure 17. Correlation of average CS and average laboratory UPV. (To convert pounds per square inch to megapascals, multiply by 0.006894757)

CS is related to the fourth power of UPV. However, even though that is true for any given concrete mixture, the exact relationship can only be established empirically by taking cores from the structure or by making cylinders from a given mixture. No analytical relationship exists between UPV and CS as it exists between UPV and modulus of elasticity. A simple linear relationship was used between the UPV and the CS, and the CS was estimated to within 20 percent of the actual value. The data are plotted in Figure 18, and the correlation coefficient calculated was 0.84. That is not a perfect correlation, nor is it a poor one. The correlation can be improved if more attention is given to obtaining good cores.

Malhotra and Carino (1991) state that the effect of moisture on the UPV of concrete is very small at room temperature (less than 1 percent). Moisture affecting the UPV in this investigation was not a significant factor since the laboratory UPV from the drier concrete agreed with the field UPV of the wetter concrete. It is likely the field concrete had a higher moisture content than the laboratory cores (no effort was made to maintain the moisture content

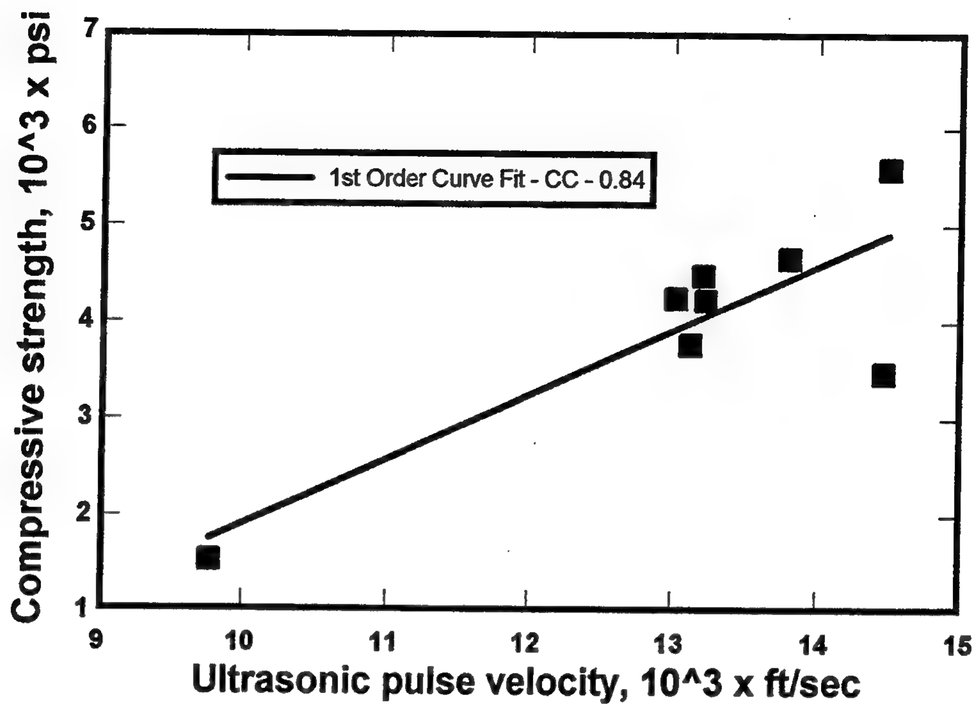

Figure 18. Correlation of average CS and field UPV. (To convert pounds per square inch to megapascals, multiply by 0.006894757)

in the cores as taken from the structure), since the tide was continually moving in and out of the marina. The correlation of field and laboratory UPV is shown in Figure 15. The correlation coefficient was 0.93. Also, there was not enough steel in the concrete to affect the UPV significantly, especially since the UPV measurements were not made in the same direction as the steel.

7 Results

RN Measurements

As previously mentioned, RN are highly dependent on the surface condition of the concrete. Therefore, an analysis of these results should theoretically be based upon a comparison of the RN for each pilaster at a given height, because the effects from the rise and fall of the tide on the surface are a function of the vertical position on each pilaster. The lower elevations of the pilasters have more marine deposits on the surface than the higher elevations, since each tide covered the lower elevations for a longer time. However, even though this constant-height measurement is important, the correlation between RN and CS was not that good for this particular investigation.

A lower RN generally corresponded with a lower field UPV value in the same locations on the pilasters, and likewise, a higher RN corresponded with a higher UPV in similar locations. There were a few exceptions in which a low UPV reading corresponded to a high RN reading. One possible explanation may be the presence of a piece of aggregate immediately beneath the surface where the RN was obtained. In this scenario, a higher average RN would be obtained than what was normally expected from a deteriorated surface. Also, the UPV reading represents the average condition of the concrete across the full width of the pilasters, and the RN does not. Also since concrete is not a homogeneous material, it is expected that some normal variation would occur between the average RN obtained at one particular location and the UPV reading in some cases. The estimation of CS with RN was not as good as the estimation of CS with UPV. The correlation coefficient was 0.61 for the RN-CS curve, while the correlation coefficient was 0.84 for the CS-UPV curve.

UPV and CS Measurements

In comparing the field and laboratory evaluations of the pilasters and the cores, the areas of the pilasters shown to be in good condition by field UPV evaluations were confirmed by laboratory CS and UPV evaluations on cores taken from those areas. As given in Table 9, UPV results of 3,658 m/sec

(12,000 ft/sec) and above may be considered an indication of good concrete. Keeping in mind that the ranges given in the table are general in nature and that variations in concrete proportioning also produce variations in these ranges, it is reasonable to assume that 3,353 m/sec (11,000 ft/sec) may indicate that this particular concrete is in good condition. All of the CS and laboratory UPV evaluations on the cores from those areas where UPV field results are 3,353 m/sec (11,000 ft/sec) and above tend to support this conclusion.

Unfortunately, the decision to obtain cores representative of both good and poor concrete quality resulted in the inability to obtain acceptable cores in areas deemed poor by UPV evaluations. However, this in itself does point to the ability of UPV measurements to identify concrete of lowest quality. From Table 10, it can be seen that in those areas where cores were unobtainable because of poor quality concrete, field UPV results measured less than 3,353 m/sec (11,000 ft/sec). In those cases, UPV results of less than 3,353 m/sec (11,000 ft/sec) appear to represent concrete of low CS.

8 Conclusions and Recommendations

Procedure to Reduce Pitfalls and Problems

The modus operandi of some technologists is (a) to drill cores and make CS measurements to evaluate the mechanical property changes in the structure when NDE measurements would be entirely satisfactory or (b) to use NDE measurements and attempt to estimate CS without a UPV-CS or RN-CS calibration on drilled cores or laboratory cylinders made from the correct mixture proportions and materials. Neither alternative is satisfactory.

NDE in the construction field is not as advanced as other industries such as the medical industry and the aerospace industry. Presently, the United States needs a standard evaluation method that blends NDE with coring. Also diagnosticians and managers need a standard evaluation method that they can follow.

This study provides an outline of the basic procedure required to conduct an NDE evaluation to estimate in situ CS. Various pitfalls, problems, complications, and successes are touched upon in the report. The correlation curve that relates the average in situ CS to the field UPV had a correlation coefficient of 0.84. Based on Mendel's method of regression analysis, the CS was predicted to within 20 percent of the actual value. It is believed that the CS of concrete can be estimated with greater accuracy if better detail is given to obtaining good cores. In cases where the mixture proportions are known for the concrete structure and the mixture components can be obtained, cylinders can be cast in the laboratory that will permit an accurate calibration to be made without drilling cores. The mixture proportion and materials were not known in this investigation.

Cost Savings

This investigation indicates that significant savings can be made by using NDE to determine the CS of concrete rather than just coring alone. Hopefully, this report will help promote the advancement of NDE for concrete structures wherever applicable by providing needed information to engineers in charge of structures and those that diagnose structures. Destructive evaluation will continue to be the method of choice for many years to come, while NDE methodology will continue to improve in the construction industry.

The investigation reported herein made use of the best of two technologies by providing a limited use of the expensive process of coring and an expanded use of UPV measurements. Minimum perforation of the structure was desirable due to the high cost, unsightly appearance, drudgery of obtaining and evaluating cores, and reduced stability of the structure from coring.

Extent of Sampling Required

It was important, at least in this setting, that three cores be taken at each location for obtaining a proper statistical average of the CS. It is believed that the nature of the deterioration will determine the number of cores needed at a location. In this instance, the cracks were discrete and spaced inches apart. Two cores could be taken adjacent to each other, and one would break into two pieces during drilling while the other remained intact. It was clear that a number of cores were needed for a proper sampling of the concrete quality. Correlation curves were plotted that showed a direct relationship between the width of the core recovered and the UPV.

Deterioration consisting of small microcracks uniformly distributed throughout the concrete may not require as many as three cores at a location. More research is needed to describe the size and number of cores that are required to provide a suitable estimate of an average CS at a given location. In this investigation, the grid pattern was a section of concrete 508 mm (20 in.) wide, the width of the pilasters, and a height of 0.3 m (1 ft) between vertical measurements. Of course, the size of the measurement grid pattern chosen as well as the type of the deterioration will also influence how many cores are required in that grid area to represent a proper statistical average. In this situation, it is clear that measurements with UPV equipment over the 508-mm (20-in.) width are much more revealing of the average condition of the concrete than the CS made on a core at only one location across that width.

It is recommended that cores be taken in a randomly distributed pattern over the concrete structure for better correlations between CS and UPV measurements while staying completely away from the visibly deteriorated areas. This should help alleviate the problem of obtaining cores from the areas of very poor condition that do not meet ASTM C 823 (ASTM 1991f)

requirements. In addition, cores should be drilled in the same direction as the field UPV measurements when possible to eliminate any errors due to the presence of anisotropy. Although a correlation curve showed that the field UPV correlated well with the average laboratory UPV (orthogonal directions) for the three cores per location in this investigation, this procedure would improve the correlation between field UPV and CS by providing measurements more representative of the condition of the concrete.

Methods of Evaluation - ASTM C 42 and C 823

Another key factor in conducting a successful investigation is that the cores should be obtained according to the standard methods of test, ASTM C 42 and C 823 (ASTM 1991b, f). A diamond drill should be used. A strict adherence to procedures for stabilizing the drill rig is recommended in that there are no lateral stresses being applied to the drill bit which could break the core. It may be necessary to increase the core diameter to have assurance that the core will not be damaged by the drilling operation. In this investigation, smaller-diameter cores were taken to miss the reinforcement steel, which had a separation distance of 102 mm (4 in.).

Rebound Hammer Not Recommended

It was not recommended that the method of ASTM C 805 (ASTM 1991e) be used to assess the condition of the seawall due to the wide differences in surface condition from the top of the pilaster to the base. The evaluation method requires that the surface be ground to a depth of 6.4 mm (1/4 in.). This would have marred the surface appearance and would not have been acceptable since the concrete is in public view. The effects of the surface condition generally decrease in proportion to the elevation. Although a manual grinding stone was used to prepare the surface of the concrete before RN evaluations were made, it was not sufficient to eliminate the surface condition. However, the grinding was performed only to a level flush with the surface of the concrete. The correlation coefficient was 0.61 with the RN evaluation and was 0.84 with the UPV evaluation.

UPV Measurements Alone

UPV measurements by themselves can provide an evaluation of the condition of the concrete without having to drill cores and develop calibration curves. However, it is not possible to predict the CS of the concrete without coring or alternatively having information about the mixture materials and proportions for calibration purposes. It is possible to provide a relative ranking of the CS for the location having the lowest value to the location having the

highest value. It is necessary to add coring to develop calibration data if the exact threshold value of UPV that separates poor CS from satisfactory CS is to be found.

It was not intended to suggest in this report that people should include coring with their UPV measurements just so they can know the CS of the concrete of a structure. The UPV method is useful in its own right to determine the location and extent of deteriorated concrete. In fact, the primary use of UPV lies in its ability to locate regions of change in the UPV property of concrete. The UPV method is quick and inexpensive, and certainly coring should not be added merely because it would be nice to have the CS. A coring program adds considerable cost to an NDE evaluation.

This investigation sought to determine the important critical parameters that will permit Government and industry to make a limited transition (the technology does not permit a complete break at this time) from destructive diagnostic methods to NDE. This report provides useful information on the successes, pitfalls, and problems encountered in applying NDE in practical field situations.

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13. ABSTRACT ***(Maximum 200 words)***

A condition evaluation conducted on a concrete seawall using (a) limited destructive evaluation and (b) extensive nondestructive evaluation (NDE) measurements demonstrated the effectiveness of the combination method (CM). The CM was used in conjunction with correlation curves and permitted the estimation of the compressive strength (CS) of the concrete from NDE measurements in locations where no coring was performed. In this study, correlation curves related the CS to two different NDE properties: ultrasonic pulse velocity (UPV) and Schmidt rebound hammer number (RN). The study demonstrated that the CS could be estimated with the UPV property to within 20 percent of its actual value by using the CM to develop correlation curves of RN versus CS and UPV versus CS. Currently, the U.S. does not have a CM measurement standard. The study discusses the circumstances where the CM can be useful and where NDE measurements alone are sufficient for a complete evaluation. This investigation also sought to determine the critical parameters that will permit Government and industry to make a limited transition—the technology does not permit a complete break at this time—from destructive diagnostic methods to NDE. In addition, the report provides useful information on the successes, pitfalls, and problems encountered in applying NDE to practical field situations.

14. SUBJECT TERMS

Combined methods	NDE of concrete	Ultrasonic pulse velocity
Correlation curves	Nondestructive evaluation system	
Estimation of compressive strength	Schmidt rebound hammer	

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